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Guo, James C.Y. (2008)"*Retrofitting Detention Basin with Water Quality Control Pool*", accepted for publication in ASCE J. of Irrigation and Drainage Engineering (Nov 2008).

RETROFITTING DETENTION BASIN WITH WATER QUALITY CONTROL POOL

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Abstract
Many detention basins built before 1990 are not equipped with stormwater quality control device. With the latest developments in low impact development (LID) for storm water management, these existing detention basins need modifications on their outlet structures to increase on-site runoff treatment and disposal. An outlet shall be designed to have, at least, three levels of release, including water quality release over 12 to 48 hours, low flow release for 10-yr event, and 100-yr high flow release. All these efforts are to aim at the full spectrum runoff treatment that is not only to capture the minor and major events but also to store micro events. Over the years, the empirical methods under different assumptions have been developed for determining the design stormwater quality control volume (WQCV). To improve the consistency in stormwater detention designs, this paper presents a mathematical model that produces the synthetic runoff-volume capture curves normalized by the local average rainfall event-depth. A runoff-volume capture curve defines the relationship between WQCV and runoff capture ratio on a long term basis. A higher runoff capture ratio requires a larger storage volume. Using the runoff capture curve as the basis, the WQCV can be consistently determined for the pre-selected runoff capture target such as 80% recommended by US EPA (1986). A case study illustrates how to retrofit an existing outfall concrete vault with a perforated plate and a micro pool for WQCV. With a three-level release control, the outfall box can have a slow release for micro events and a fast release for extreme events. This procedure has been recommended for designing a new basin and retrofitting an existing for the metro Denver area, Colorado. Details can be found in the UD-DETENTION computer model available at **www.UDFCD.org** at no cost to download.

Key Words: Stormwater, Water Quality, Detention, Retention, Runoff, Rainfall

INTRODUCTION

A flood control detention basin (FCDB) is often constructed to control extreme events, including a lower layer for the minor detention volume and the upper layer for the major event control (USWDCM 2001). A minor event is often defined to be 5 to 10-yr storm and the major event is referred to as 50 to 100-yr storms. Starting in early 1990's, the Federal Clean Water Act has shifted the focus of stormwater detention design in the United States from flood mitigation to stormwater quality control. As an alternative to the traditional approach developed for extreme events, a storm water quality control basin (WQCB) is recommended to capture the micro events at the magnitude of the first flush volume (US EPA 1986 and 1983). Often, the outfall system for a FCDB constructed to mitigate the minor and major events is "too big" for micro events. In other words, the small runoff flows are always to trickle through the outlet openings without any storage effect. As a result, it is an increasing concern about how to retrofit an existing FCDB to provide stormwater quality enhancement to the frequent events.

A WQCB can be a micro pool or an infiltrating bed at the low point in a FCDB. Of course, the existing FCDB's outlet system needs to be modified with the ability of full spectrum runoff control (Urbonas and Wulliman 2005). For instance, a perforated plate and riser can be added to the outlet system. The slow outflow through a perforated plate creates the detention effect and increases the residence time for solid removal. A WQCB must be shaped to capture the first street flush runoff volume. There are many recommendations that calculate the design WQCV using empirical formulas. For instance, Montgomery County in the State of Maryland (Maryland, 1986) requires the basin storage volume to be equal to 12.3

mm (0.5 inch) of runoff depth from the tributary watershed while the Maryland Water Resources Administration suggests a volume equal to 2.5 times the runoff volume generated by the mean storm. Roesner et al. (1996), based on a review of runoff distribution from different areas of United States, suggested that a cost effective basin size should have a volume equal to runoff volume from a storm of 4-month return period. Along with these efforts, this paper presents an attempt to construct the localized synthetic runoff capture curve for WQCB designs. A runoff capture curve defines the relationship between storage volume and runoff capture rate. It provides a consistent basis to determine the WQCV to size a WQCB.

RAINFALL EVENT-DEPTH DISTRIBUTION

Derivation of runoff capture curves begins with the understanding of general characteristics of complete rainfall data series. Although drainage and flood control engineers often consider the 2-year storm event a small storm, it, in fact, produces runoff greater than 95 percent of the events that may occur in an urban catchment (Guo and Urbonas, 1996). Figure 1 shows the frequency distribution of rainfall event-depths observed over 40 years in the Denver area, Colorado. 94.1% of the observed storm events had a precipitation depth less than 0.95 inch that is equal to the one-hour, two-year rainfall depth in Denver, Colorado. Although the skewness of the rainfall frequency distribution varies from one region to another, it can be concluded that micro events dominate a long-term rainfall record.

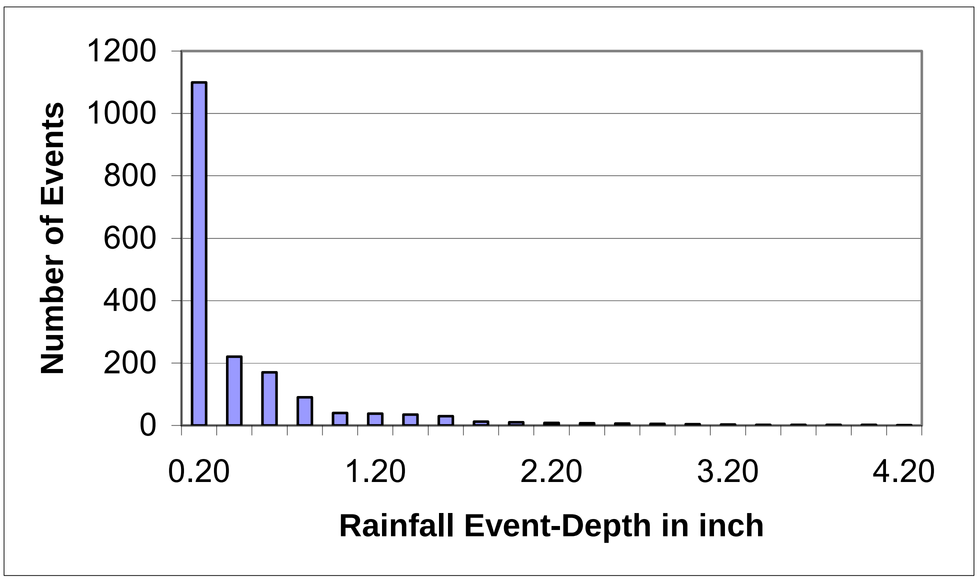

Figure 1 Rainfall Depth Distribution in Denver Area, Colorado

There are many recommendations on modeling the distributions of complete rainfall data series, such as exponential distribution (Bedient and Huber, 1992), one-parameter Poisson distribution (Wanielista and Yousef, 1993), and two-parameter model of Gamma distribution (Woolhiser and Pegram, 1979). In this study, the one-parameter exponential distribution is adopted to fit the frequency distribution of rainfall event depths (Guo 2002). The exponential distribution is described as:

$$f(D) = \frac{1}{D_m} e^{\frac{-D}{D_m}} \tag{1}$$

in which *f(D)* = frequency of rainfall event-depth, *D*, and D_m = average rainfall event-depth. The values of D_m can be found elsewhere (US EPA 1989, Guo 2002). Integration of Eq 1 represents the cumulative probability distribution as:

$$P_D(0 \le d \le D) = 1 - e^{\frac{-D}{D_m}} \quad (2)$$

Eq 2 depicts the distribution of non-exceedence probability, P_D, that represents the chance to have an event-depth, *d*, not to exceed the design depth, *D*. The corresponding exceedence probability is

$$P_D(D \le d \le \infty) = e^{\frac{-D}{D_m}} \qquad (3)$$

Both Eq's 2 and 3 are derived for the distribution of rainfall depths. A WQCB is designed to intercept runoff volumes, not rainfall event-depths. Therefore, it is necessary to convert Eq's 2 and 3 into the runoff volume distribution.

RUNOFF CAPTURE CURVE

For convenience, WQCV is expressed in unit depth per watershed such as mm or inch per watershed. Since the purpose of this study is to determine the runoff volume to be captured, therefore only runoff-producing events will be considered for analyses. As recommended, an incipient runoff depth of 2.5 mm is introduced to filter out extremely small rainfall events (Guo and Urbonas in 1996, US EPA in 1989). The WQCV can then be related to its design rainfall depth as:

$$V_o = C(D - D_i) \qquad (4)$$

in which V_o = WQCV in mm per watershed, *C*= runoff coefficient, *D* = design rainfall depth, and D_i = incipient runoff depth. Re-arranging Eq 4 yields:

$$\frac{D}{D_m} = \frac{V_o}{CD_m} + \frac{D_i}{D_m} \qquad (5)$$

Substituting Eq 5 into Eq 3 yields

$$P_D(0 \le V \le V_o) = P_D(0 \le d \le D) = 1 - e^{-(\frac{D_i}{D_m} + \frac{V_o}{CD_m})} \qquad (6)$$

in which $P_D(0 \le V \le V_o)$ = probability to have an event that produces a runoff depth less than V_o or P_D is the runoff capture ratio between zero and unity. In this study, Eq 6 is termed the synthetic runoff capture curve that is normalized by local average rainfall event-depth, runoff coefficient, and runoff incipient depth. Re-arranging Eq 6 yields:

$$C_v = 1 - ke^{\frac{-V_o}{CD_m}} \qquad (7)$$

$$k = e^{\frac{-D_i}{D_m}} \qquad (8)$$

In which *k* = constant determined by incipient runoff depth and C_v = runoff volume capture rate between *k* and unity. The value of *k* represents the watershed natural depression capacity. Figure 2 presents a set of generalized runoff capture curves produced using Eq 6 with runoff coefficients of 0.4, 0.6, 0.8, 0.9 and 1.0. It is noticed that the curvature of runoff capture curve increases when the runoff coefficient decreases. The runoff capture curve becomes almost a linear response between rainfall depth and runoff amount when *C=1.0*. This tendency reflects the fact that the higher the imperviousness in a watershed, the less the surface depression and detention. As a result, the response of a watershed to rainfall is quick and direct

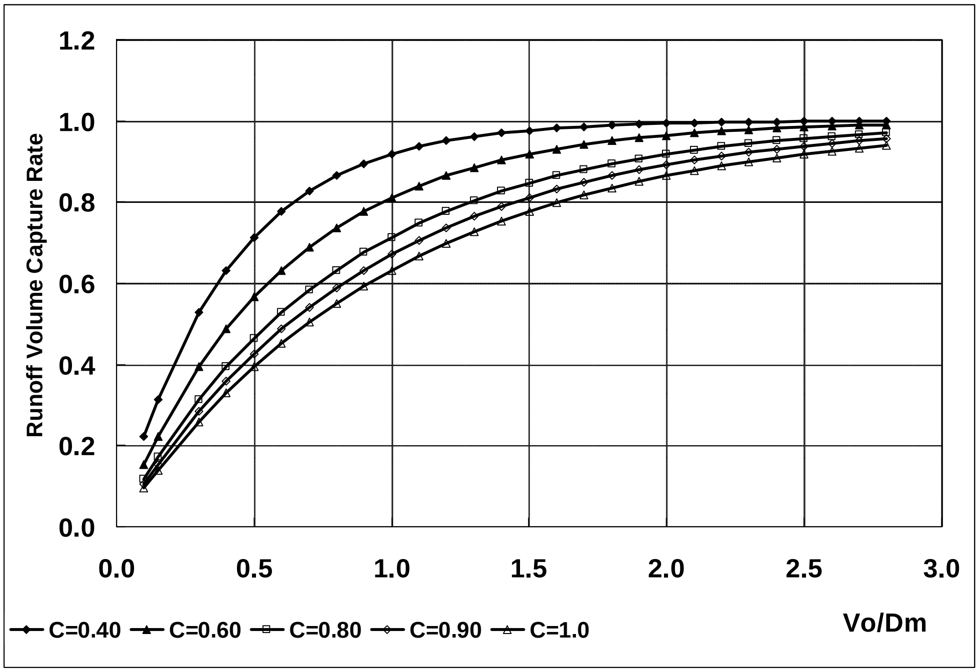

Figure 2 Normalized Runoff Capture Curves for Various Runoff Coefficients

The runoff capture curve provides a consistent basis to determine the WQCV for the pre-selected runoff capture rate as a target. In current practice, the runoff capture curve at a basin site is generated by long-term runoff simulations using the computer model: EPA SWMM 5 (Rossman 2005). Often, such a lengthy data process is not practical. Indeed, it is imperative that the design methodology be improved by the fundamental understanding of the distribution of runoff-producing rainfall series. Eq 7 provides a set of synthetic runoff capture curves using the local average rainfall event-depth as the base parameter to represent the site characteristics and the runoff coefficient as the floating parameter to represent the watershed development. For a specified runoff coefficient, the runoff capture rate increases when the WQCV increases. The runoff coefficient also plays an important role in the decay factor in Eq 7. As expected, the runoff capture rate tends to be lower for a paved area than that for a pervious area.

DESIGN SCHEMATICS

To illustrate the design procedure, the existing DB located in Denver, Colorado is employed as an example. The DB has been constructed to control the 10- and 100-yr outflows. The 10- and 100-yr water depths are 4 and 8 feet above the basin floor respectively. The existing outlet system for this DB is a 2 x 2-ft concrete box culvert that is 100-ft long on a slope of 2%. This DB needs to be modified for full spectrum runoff control. As illustrated in Figure 3, the tasks include: (1) determination of WQCV (2) sizing the micro pool (3) design of the concrete vault to be installed in front of the existing culvert. This concrete vault shall be equipped with a perforated plate for micro event release, a vertical orifice for 10-yr release, and a top horizontal grate for 100-yr release.

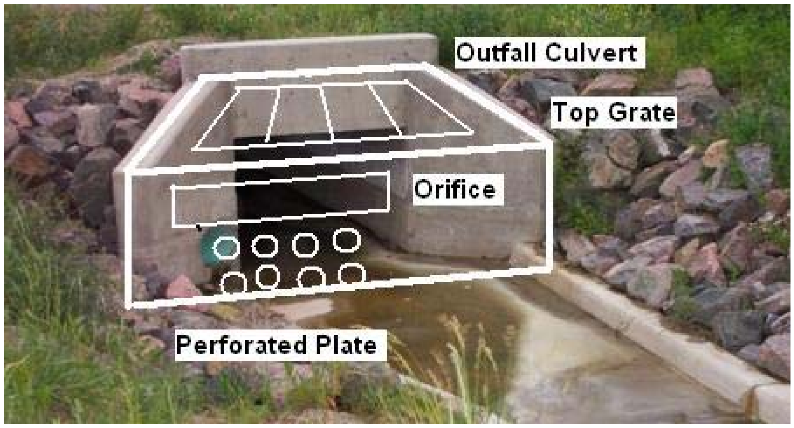

Figure 3 Example of Retrofitted Outlet

For this case, a WQCB is to be added to the bottom of the existing DB. The task is to retrofit the existing outlet to achieve a runoff capture rate at 80% or 80% of smaller events will be completed stored in the proposed WQCB. The tributary watershed to this DB has a drainage area of 50 acres (20.0 ha) and a runoff coefficient of 0.6. As reported (US EPA 1989), at the basin site, the average rainfall event-depth is 0.41 inch (10.4 mm) and the incipient runoff depth is 0.1 inch (2.54 mm). Aided by Eq's 6, 7 and 8, the on-site runoff volume capture curve is derived as:

$$C_v = 1 - e^{-(\frac{0.1}{0.41} + \frac{V_o}{0.6 \times 0.41})} = 1 - 0.78 e^{-4.2 V_o} \tag{10}$$

To target a runoff capture rate at 80%, the required WQCV, V_o in Eq 10, is found to be 0.32 inch (8.1 mm) per watershed or *WQCV = 1.3 acre-ft* (1604 m^3) for a tributary area of 50 acres (20 ha). Considering a micro pool of 2-ft (0.6 m) deep, the average cross section area for the micro pool is calculated to be 0.65 acres (2632 m^2).

To retrofit the outfall system for the design WQCV, a concrete vault as shown in Figure 4 is added to the entrance of the existing box culvert. Considering a drain time of 24 hours, the perforated plate, 3-ft (91 cm) wide and 2-ft (60 cm) high with 5-row and 5-column one-inch holes, is selected for this DB. The perforated plate is protected by a trash screen. This perforated plate creates a pool of 2-ft (60 cm) deep for the design WQCV. Immediately above the perforated plate, a vertical orifice, 2-ft (60 cm) high and 3-ft (91 cm) wide, is installed on the vault to control the extreme-event releases. For this case, the top horizontal grate is not needed.

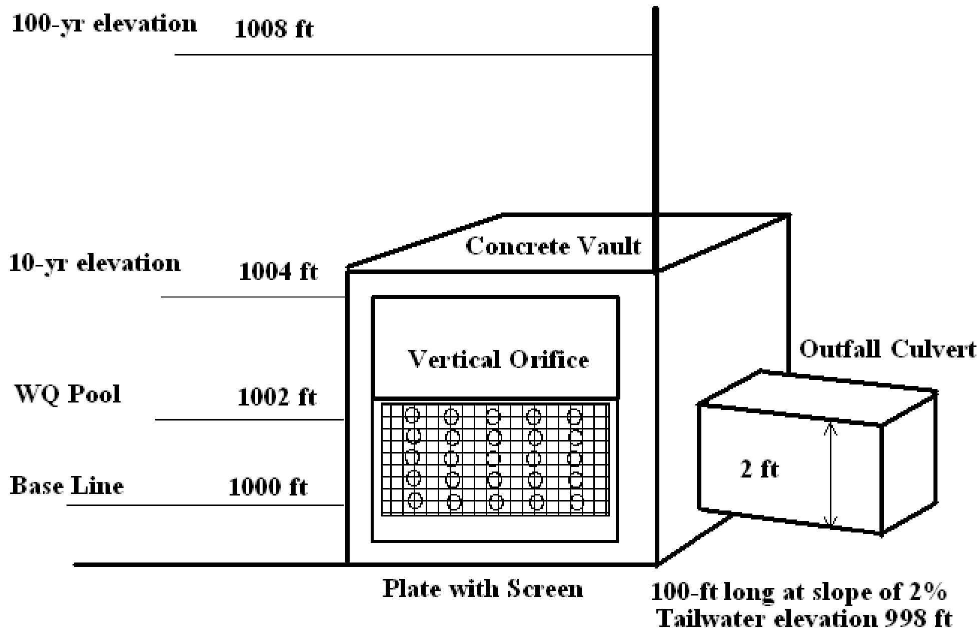

Figure 4 Retrofitted Outlet System Using Concrete Vault

The concrete vault is connected to the existing box culvert. The total flow collection capacity into the concrete vault is the sum of the inflows through the perforated plate and the vertical orifice. For a specified water surface elevation in the basin, the flow through the vertical orifice is calculated as:

$$Q_o = C_o A_o \sqrt{2g(H - h_o)} \tag{11}$$

In which Q_o = vertical orifice flow, C_o = orifice coefficient such as 0.65, A_o= *flow area,* h_o = central elevation of orifice opening area, g = gravitational acceleration, *and* H = water surface elevation in DB. Similarly, the flow through a row of holes on the perforated plate is determined as:

$$Q_p = C_o N A_p \sqrt{2g(H - h_p)} \tag{12}$$

In which Q_p = flow collected by the holes with their center elevation at h_p, N= number of holes, and A_p= unit hole area. For the given water surface elevation, H, the discharge capacity through the outfall culvert is calculated as:

$$Q_c = A_c \sqrt{\frac{1}{K+1}} \sqrt{2g\left(H - h_t\right)} \tag{13}$$

In which Q_c = culvert discharge capacity, A_c = culvert opening area, h_t = tailwater elevation, and K = sum of loss coefficients determined as:

$$K = K_e + K_x + K_b + 29 \frac{n^2 L}{R^{4/3}} \tag{14}$$

In which K_e = entrance loss coefficient, K_x = exit loss coefficient, K_b = bend loss coefficient**,** n = Manning's roughness coefficient L = culvert length, and R= hydraulic radius. For the given water surface elevation, the outflow from the DB is dictated by the smaller one between the collection capacity into the concrete vault and the discharge capacity through the outfall box culvert as:

$$Q(H) = \min(Q_o + Q_p, Q_c) \qquad (15)$$

Where Q(H) = outflow from the basin under water surface elevation, H. The outflow detailed calculation procedure can be found elsewhere (Mays 2001).

Table 1 summarizes the calculations of the two stage-outflow curves, one for the existing box culvert and another for the new concrete vault. The WQCV below 2 feet (60 cm) is controlled by the slow release through the perforated plate while the 10- and 100-yr storage volumes are still controlled by the outfall box culvert. As shown in Figure 5, the retrofitted stage-outflow curve is merged into the existing for water depths above the WQCV pool. The ultimate goal for a detention process is to preserve the watershed regime or the slow release through the perforated plate can simulate the base flow in the receiving stream, and the 10- and 100-yr releases through the orifice and the culvert can mimic the pre-development condition.

Water	Existing Condition			Retrofitted Condition				
Depth ft	Collection Capacity cfs	Discharge Capacity cfs	Existing Outflow cfs	Vertical Orifice cfs	Perforated Plate cfs	Collection Capacity cfs	Discharge Capacity cfs	Retrofitted Outflow cfs
	Eq 11	Eq 13	Eq 15	Eq 11	Eq 12	Eq 11+Eq 12	Eq 13	Eq 15
0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.0	6.79	16.19	6.79	0.00	0.24	0.24	16.19	0.24
2.0	23.18	22.89	22.89	0.00	0.62	0.62	22.89	0.62
3.0	32.77	28.04	28.04	9.93	0.99	10.92	28.04	10.92
4.0	40.14	32.37	32.37	33.91	1.23	35.14	32.37	32.37
5.0	46.35	36.20	36.20	47.95	1.41	49.36	36.20	36.20
6.0	51.82	39.65	39.65	58.73	1.60	60.33	39.65	39.65
7.0	56.77	42.83	42.83	67.81	1.75	69.56	42.83	42.83
8.0	61.32	45.78	45.78	75.81	1.89	77.70	45.78	45.78

Note: K_e=0.5, K_x=1.0, K_b=0, n=0.015, and L=100 ft

Table 1 Stage-Outflow Curves for Existing and Retrofitted Conditions.

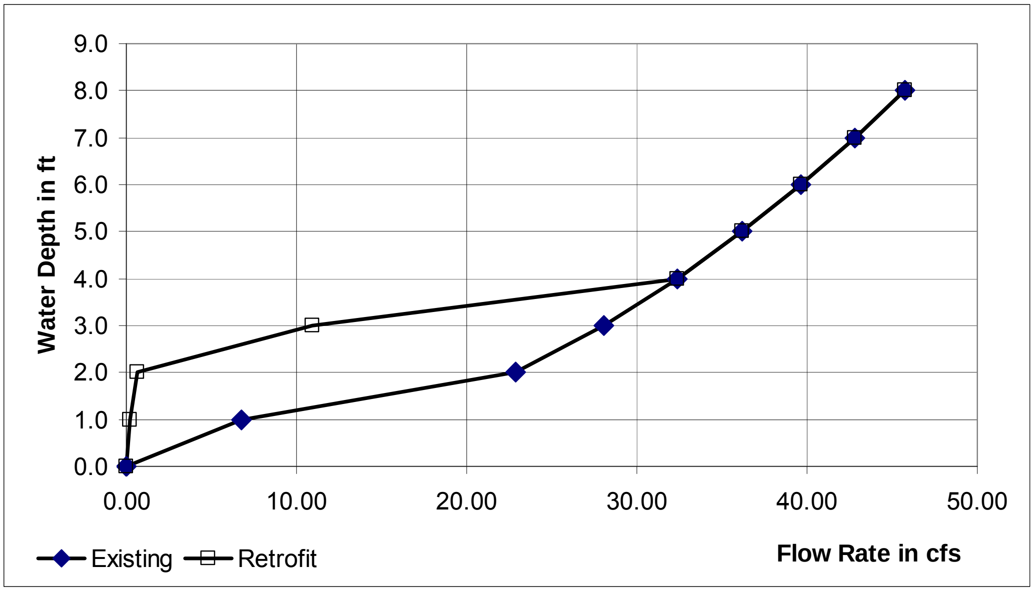

Figure 5 Existing and Retrofitted Stage-Outflow Curves for Detention Basin

CONCLUSIONS

Urban hydrology has been developed with an emphasis on extreme events, i.e. minor and major storms. The flood-frequency curve defines the relationship between flood magnitude and exceedance probability. With a pre-selected level of protection, the design capacity can be determined for a flood control facility. Under the mandate of Federal Clean Water Act, the frequency-based approach is found no longer suitable for WQCB designs. This study presents a methodology to produce the on-site synthetic runoff-volume capture curves by which the WQCV can be consistently calculated for the pre-selected runoff capture rate. The synthetic runoff-volume capture curve is normalized by the local average rainfall event-depth and the runoff coefficient representing the level of watershed development. The required prior knowledge of average rainfall event-depth for the United States continent can be found elsewhere (US EPA report 1989, Guo 2002).

To retrofit an existing detention basin, a micro pool is added to the bottom of the basin for stormwater quality enhancement. A perforated plate or riser is recommended to create an extended, slow release for the design WQCV. The operation of the micro pool is then incorporated into the outlet system using a concrete vault with vertical and horizontal orifices. The stage-outflow curve for the retrofitted detention basin is determined between the flow collection capacity of the new concrete vault and the discharge capacity of the outfall culvert, whichever is smaller. This design procedure has been coded and documented in the compute model, UD-DETENTION (USWDCM 2001), that can be downloaded from the website: WWW.UDFCD.ORG at no cost.

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APPENDIX II. Notations

A_c = culvert opening area
A_o= *flow area*
A_p= unit hole area on perforated plate
C= runoff coefficient
C_o = orifice coefficient such as 0.65
C_v = runoff volume capture rate
D = design rainfall depth

D_i = incipient runoff depth
D_m = average rainfall event-depth
$f(D)$ = frequency of rainfall event-depth
g = gravitational acceleration
H = water surface elevation in basin
h_o = central elevation of orifice opening area
h_t = tailwater elevation
k = constant determined by incipient runoff depth
K = sum of loss coefficients
K_e = entrance loss coefficient
K_b = bend loss coefficient
K_x = exit loss coefficient
L = culvert length
N= number of holes
n = Manning's roughness coefficient
P_D $(0 \leq d \leq D)$ = probability to have an event-depth, *d*, not to exceed the design depth, *D*
$P_D(0 \leq V \leq V_o)$ = probability to have an event that produces a runoff depth less than V_o
R= hydraulic radius
Q(H) = outflow from the basin under water surface elevation, H.
Q_c = culvert discharge capacity
Q_o = vertical orifice flow
Q_p = flow collected by the holes with their center elevation at h_p
V_o = WQCV in mm per watershed